WYOMING DEPARTMENT OF ENVIRONMENTAL QUALITY
SOLID AND HAZARDOUS WASTE DIVISION

SOLID WASTE GUIDELINE # 18

Solid Waste Containment Facility Slope Stability and Seismic Deformation Evaluation

1.0 Introduction and Background

The importance of slope stability on the operational performance and long-term behavior of solid waste containment facilities is underscored by a number of component / slope failures in waste containment facilities in the United States in recent years.

Nearly 90% of the State of Wyoming falls within a Seismic Impact Zone as defined by the USEPA under 40 CFR 258.14(b) (1) as an area with a ten percent or greater probability that the maximum horizontal acceleration in lithified earth material, expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10g in 250 years. Maximum horizontal acceleration in lithified earth material means the maximum expected acceleration depicted on a seismic hazard map, with a 90 percent or greater probability that the acceleration will not be exceeded in 250 years, or the maximum expected horizontal acceleration based on a site specific seismic risk assessment.

Lithified earth material means all rock, including all naturally occurring and naturally formed aggregates or masses of minerals or small particles or older rock, that formed by crystallization of magma or by induration of loose sediments. This term does not include man-made materials, such as fill, concrete and asphalt or unconsolidated earth materials, soil or regolith lying at or near the earth surface.

2.0 Regulatory Considerations

Existing landfills with new cells or horizontal expansions of area fills are restricted from locating these facilities in a seismic impact zone under Wyoming Solid Waste Rules and Regulations Chapter 2, Section 3 (b)(i)(E), unless all containment structures, including liners, leachate collection systems, and surface water control systems, are designed to resist the maximum horizontal acceleration in the lithified earth material for the site.

New facilities may not be located within 200 feet of a fault that has had displacement in Holocene time according to Chapter 2, Section 3 (xiii). Further, new facilities are required to show stability of their containment systems if they are located in a seismic impact zone, e.g. vertical, lateral or overlay expansions. Existing facilities are not allowed to locate within 200 feet of a Holocene fault.

Analysis of the static and seismic stability of the containment systems at the facility must be evaluated for the following activities:

- Cell / Liner construction
- Land filling operations
- Lateral and Vertical Expansion or Overlay
- Relocation of old waste; especially when the waste consists primarily of soil
- Rapid Waste re-grading for closure purposes,
- Final Closure / cap and letdown construction, and
- Leachate containment impoundments.

Chapter 2, Sect. 3. (b)(i) (F) states "Existing facilities, new landfill cells at existing facilities, and horizontal expansions of area fills at existing facilities, shall not be located in an unstable area unless the owner has demonstrated to the administrator that engineering measures have been incorporated into the facility's, cell's, or area fill's design to ensure that the integrity of the structural components of the facility, cell, or area fill will not be disrupted.

The demonstration must consider:

- On-site or local soil conditions that may result in significant differential settling;
- On-site or local geologic or geomorphologic features; and
- On-site or local human-made features or events (both surface and subsurface)."

3.0 Technical Content of Seismic Design Submittal

The evaluation of static and seismic stability should consider at a minimum the following factors:

- Appropriate selection of the potential failure modes, geometry, stability methods, shear strength parameters, laboratory testing methodology, parameters, pore pressure conditions, and construction materials.
- An appropriate CQC/CQA plan to ensure that construction is performed according to the engineered design.
- Limitation of toe excavations and implementation of a construction management program to ensure expansions proceed according to an appropriate filling schedule.
- Filling schedules that are developed consistent with the stability analysis and development of monitoring techniques that prevent overfilling.
- Contingency plans in the event of changed conditions during construction and filling, (excessive rainfall, unexpected foundation conditions or construction delays, etc.)
- Waste diversion plans so slopes are not over built due to construction delays or the shortage of permitted air space.

Potential failure modes should include at a minimum:

- The critical cross section through the particular containment system (waste liner and leachate collections system, or storm water containment system) during the following three phases;
    - excavation and construction,
    - filling operations, and

- closure / post-closure.

- The critical cross-section for each of these conditions for each containment system is developed by first selecting the worst case factors that affect the stability of the slope(s). These cross sections are then analyzed using a limit equilibrium slope stability computation method to estimate the factor of safety. The cross-section slip surface that yields the lowest factor of safety is referred to as the critical slip surface.

Factors that affect the factor of safety and therefore the stability of the slope include:

- Slope geometry, height, inclination, surcharges and other incidental loading such as heavy equipment of temporary stockpiling (added driving forces); toe berms or buttresses (added resisting forces).
- Shear strength of interfaces, waste, cover materials, foundation materials (bearing failure or excessive settlement) and the effects of seismicity (liquefaction)
- Pore-water pressure / seepage forces
- Gas Pressures
- Loading conditions including driving forces (e.g., unit weight of waste, compaction and composition of waste), addition of moisture; and pre- and post-closure conditions (e.g., seismic events, settlement, and expansion).
- Settlement caused by the landfill (e.g., increase in unit weight and infiltration); and the foundation material(s) (e.g., soils and organics).

3.1. Slope Stability Analysis

A two dimensional numeric computational method should be employed in evaluating the stability of the critical cross-section. The General Limit Equilibrium (GLE) theory should be used as the basis of the computational method as this method can accommodate circular and non-circular slip surfaces. Most of the two-dimensional limit equilibrium methods used involve passing a slip surface through the earthen mass and dividing the inscribed portion into vertical slices. The slip surface may be circular, composite (i.e., combination of circular and linear portions) or consist of any shape defined by a series of straight lines (i.e., fully specified slip surface). For each slip surface increment, whether it is through several interfaces or one material, there will be a factor of safety (FOS) equal to the sum of the resisting forces, shear stresses, or moments of each slice divided by the sum of the opposing / driving forces, shear stresses, or moments of each slice. All equilibrium methodologies make the following assumptions:

- The shear strength is fully mobilized along the failure surface, and
- The normal force acts at the center of the vertical slice base.

According to Duncan[1] limit equilibrium stability methods which meet all conditions of equilibrium, i.e., horizontal and vertical force equilibrium, individual slice moment equilibrium and overall moment equilibrium, result in the most accurate FOS. The following table (Table #1) shows the static equilibrium conditions satisfied by various equilibrium methods. Methods that satisfy all equilibrium conditions, e.g., Spencer, Morgenstern-Price or Sharma will produce factors of safety less than 5 percent error.[2]

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[1] Duncan, J.M. 1992. "State-of-the-Art: Static Stability and Deformation Analysis." Proc. Spec. Conf. on performance and Stability of Slopes and Embankments – II, American Society of Civil Engineers, Geotechnical Special Publication No. 31, Vol. 1, Berkeley, CA, pp. 222-266.

[2] MDNR and Stark, T.D. 1998. "*Draft Technical Guidance Document on Static and Seismic Slope Stability for Solid Waste Containment Facilities*," Unpublished Draft, pg13.

Table 1.

Conditions of Static Equilibrium Satisfied by Various Limit Equilibrium Methods Force Equilibrium[3]			
Method	1st Direction (vertical)	2nd Direction (horizontal)	Moment Equilibrium
Ordinary or Fellenius[4]	Yes	No	Yes
Bishop's Simplified[5]	Yes	No	Yes
Janbu's Simplified	Yes	Yes	No
Janbu's Generalized[6]	Yes	Yes	No
Spencer[7]	Yes	Yes	Yes
Morgenstern-Price[8]	Yes	Yes	Yes*
GLE	Yes	Yes	Yes
Corps of Engineers	Yes	Yes	No
Lowe-Karafiath	Yes	Yes	No
Sharma[9]	Yes	Yes	Yes
* Moment equilibrium on individual slice is used to calculate inter-slice shear forces			

The main effort in analysis should therefore be spent defining the slope geometry, realistic failure modes / surfaces, pore water pressure ranges, unit weights and shear strength after selecting a suitable computational method.

Numeric Models – Numeric modeling can be performed accurately by commercially available computer programs. These programs should be able to perform circular and non-circular slip surface analysis using either Spencer,

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[3]Krahn, J. 2004. *Stability Modeling with Slope /W – An Engineering Methodology*, Geo-Slope International, Ltd. 2004, pp. 375, Table 12-3.
[4] Fellenius, W., 1936. Calculation of the Stability of Earth Dams. Proceedings of the Second Congress of Large Dams, Vol. 4, pp. 445-463.
[5] Bishop, A. W. 1955. "The Use of the Slip Circle in Stability Analysis of Slopes," Geotechnique, Vol. V, No. 1, pp.7-17.
[6] Janbu. N. 1957. "Earth Pressures and Bearing Capacity of Calculations by Generalized Procedure of Slices," Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, London, pp.207-212.
[7] Spencer, E. 1967. "A Method of Analysis of Embankments assuming Parallel Interslice Forces," Geotechnique, Vol. 17, No. 1, March 1967, pp. 11-26.
[8] Morgenstern, N. R., and Price, V. E., 1965. "The Analysis of the Stability of General Slip Surfaces," Geotechnique, Vol. 15, No. 1,, pp. 79-93.
[9] Sharma, S. K., 1973. "Stability Analysis of Embankments and Slopes," Geotechnique, Vol. 23, No. 3, pp. 423-433.

Morgenstern-Price or Sharma methodologies as described above. Examples of these programs are XSTABL, UTEXAS2, UTEXAS3, SLOPE/W Version 5 or SLOPE/W 2004 GeoStudio™. Each of these programs has the capability of producing graphic and numeric output of the input conditions and parameters, as will as of the technical results. The design engineer should provide sufficient detail of the analysis that will enable the Department of Environmental Quality (Department) to follow the computations in hard copy and preferably even allow duplication of the numeric results by supplying an electronic input file that can be run (but not modified) on a viewer licensed version of the software used in the analysis.

3.2. Factors of Safety

- **Static Slope Stability Analysis – (SSSA)** - Use a minimum FOS for static conditions of 1.50. The FOS obtained using a limit equilibrium method is defined as that factor by which the shear strength of the soil must be reduced to bring the mass of soil into as state of limiting equilibrium along the selected slip surface. Therefore any FOS greater than unity (1.0) would be considered to be "safe from failure". The greater the value, the less likely that failure would occur.
- **Pseudo-Seismic Slope Stability Analysis – (PSSSA)** - Use a minimum FOS of 1.10. The stability evaluation of the critical slope may be evaluated as if it were under a seismic loading event, through the computation of a pseudo-static FOS for the critical slip surface determined from the static limit equilibrium analysis. This pseudo-seismic analysis is accomplished using a seismic coefficient (K). The Seismic Coefficient (K_x or K_y) equals the fraction of the weight of the potential failure mass that is applied as a horizontal force to the centroid of the mass in a pseudo-static limit equilibrium stability analysis. The seismic coefficient is typically specified as a fraction of the free-field Peak Horizontal Ground Acceleration (PHGA) of the design earthquake divided by the acceleration of gravity [K = PHGA/g], where the acceleration of gravity is 32.2 ft/sec^{2}and the PHGA is the Maximum (or peak) Horizontal Acceleration from the regulatory maximum earthquake event (A_{max} or MHA). A typical A_{max} is a multiple of the peak horizontal

bedrock acceleration, which is estimated from the USGS Map.[10] Development of the appropriate PHGA for a site is dependent upon the presence of unconsolidated foundational soils. See the following section on the development of the PHGA and the need for a Seismic Site Response Analysis.

One of the greatest drawbacks of the pseudo-static analysis is that it does not allow the computation of the magnitude of displacement in a slope after shaking and as a result the method cannot be used to determine whether the sum of all displacements will cause structural damage. Therefore, employing a Factor of Safety threshold of 1.10 provides a degree of certainty that the slope is sufficiently stable for the design seismic event to assure that no permanent deformation will occur because the yield acceleration will not be exceeded. The yield acceleration (A_y) is the acceleration that results in the pseudo-static Factor of Safety being reduced to unity. If the pseudo-static factor of safety never falls to unity for the acceleration of the center of the critical static failure surface induced by the design earthquake then the yield acceleration will always be greater than the design acceleration. If so, no permanent deformations will be induced. Thus a pseudo-static factor of safety greater than unity is used. That value has been set at 1.10 as previously noted above.

Conversely, if the pseudo-static slope stability analysis factor of safety is less than unity, permanent deformation may be induced in the waste containment envelope and a Seismic Deformation Analysis will be required to assure that the design is sufficient to limit the magnitude of deformation of various components is below the maximum thresholds described below. As a precaution to provide a degree of assurance that no permanent deformation will occur during the design event, a Seismic Deformation Analysis if the Pseudo-Static FOS is less than 1.10.

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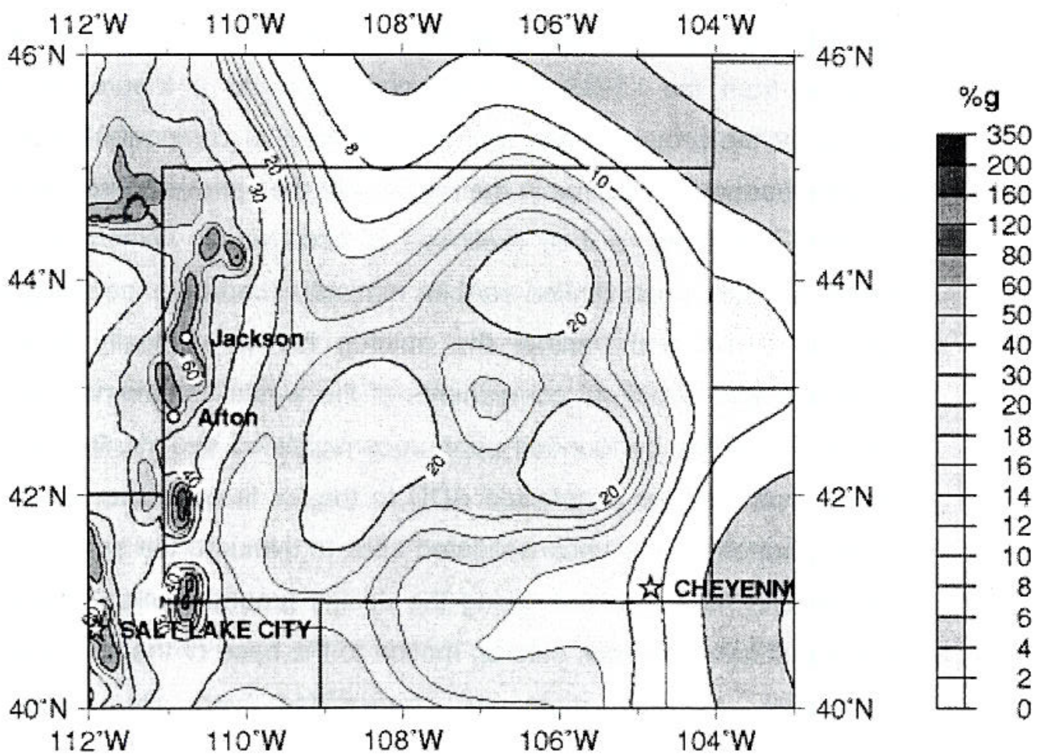
[10] Algermissen, S.T., Perkins, D.M., Thenhaus, P.C., Hanson, S.L., and Bender, B.L., 1990. "Probabilistic Earthquake Acceleration and Velocity Maps for the United States and Puerto Rico." U.S. Geological Survey, Open-File Services Section, Branch of Distribution, Box 25425, Federal Center, Denver, CO. 80225, Misc. Field Studies Map MF-2120.

3.3. Peak Horizontal Ground Acceleration

The Peak Horizontal Ground Acceleration, (PHGA, Amax or MHA), may be determined using Table 1. It is recommended that the designer determine the PHGA using the Lat/Long plotting capability found at the Seismic Hazard Mapping Project map web site[11]. Since there is significant sensitivity in the value of this factor based on relatively small changes in position in much of the State of Wyoming, it is recommended that the designer determine the appropriate PHGA at the extreme corners (limits) of the waste containment boundary and determine the maximum PHGA for the footprint rounded to 3 significant figures. This value is expressed as a % of Gravity and is also known as the Seismic Coefficient (K).

Figure 1.



Peak Acceleration (%g) with 2% Probability of Exceedance in 50 Years
site: NEHRP B-C boundary
National Seismic Hazard Mapping Project

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[11] http://eqhazmaps.usgs.gov/index.html or http://earthquake.usgs.gov/hazmaps/

The Seismic Hazard Mapping Project Map[12] shown here is the 1996 iteration of the above referenced map and is the typical source of the horizontal seismic coefficient – K. The USGS map presents contours of the horizontal bedrock acceleration with the 10% probability of Exceedance in 250 years earthquake event. Figure 1 shows the equivalent with a 2% probability of this occurrence in 50 years. When the PHGA is greater than or equal to 0.10g the site is by definition located within a Seismic Impact Zone according to USEPA under 40 CFR 258.14(b)(1). If a site is located in a Seismic Impact Zone the designer is required to evaluate the seismic stability of all components of the waste containment envelope(s) at the facility.

The seismic coefficient (K) multiplied by the acceleration of gravity is the Maximum Horizontal Earthquake Acceleration (MHA) (also known as the Maximum Peak Acceleration (A_{max}) or noted earlier as the PHGA) acting on the center of gravity of the slide mass. The probabilistic value of PHBA obtained from the USGS website corresponds to a theoretical bedrock outcrop at the proposed site and is composed of components from every possible earthquake source in the region with the various components being weighted according to their likelihood of occurrence. Where a facility is directly founded upon lithified earthen materials (bedrock) the designer may use this PHBA and transfer the outcrop motion vertically through the unconsolidated structural components of the waste containment envelope. Should the facility be founded upon unconsolidated sediments, the outcrop motion must first be transmitted ADD to the, or through the (whichever is correct) bottom of the unconsolidated soils to evaluate the seismic shaking that the site will encounter during the design ground motion. Here again, after transferring the rock outcrop motion to the base of the soil column, the earthquake shaking must be transmitted vertically through the

[12] http://eqhazmaps.usgs.gov/index.html or http://earthquake.usgs.gov/hazmaps/ Peak Acceleration (%g) with 2% Probability of Exceedance in 50 Years – U.S. Geological Survey National Seismic Hazard Mapping Project site: NEHRP B-C boundary Nov 1996.

unconsolidated sediments to the bottom of the proposed landfill site and then into the unconsolidated waste containment components themselves.

3.4. **Seismic Site Response Analysis** – A seismic site response analysis transfers the synthetic bedrock acceleration time histories (and peak horizontal ground motion) through the overlying unconsolidated deposits to estimate the level of shaking that will occur at the base of the liner and at the final cover system. This analysis may be performed using one of several numeric models available in the public domain, namely SHAKE (Schnabel et al, 1972), SHAKE91 (Idriss and Sun 1992), or DEEPSOIL v2.6 (Hashash et al. 2006).

Time histories, not probabilistic sums of Peak acceleration values, are used in DEEPSOIL because the response of the unconsolidated sediments in this one-dimensional program is controlled by the entire frequency content of the earthquake motion not just peak acceleration. The engineering properties required for the site response analysis are unit weight, shear wave velocity, and shear modulus and damping degradation relationships. However, shear modulus and damping degradation relationships are not required for the bedrock material because the bedrock in a site response analysis is assumed to behave elastically.

3.5. **Seismic Deformation Analysis** –Prediction of the performance of a waste mass under normal gravitational and seismic loading remains an inexact science. Although technology, understanding and computational abilities are continuously advancing, precise prediction of the properties of the waste mass, underlying foundation, and magnitude, frequency and duration of the design seismic event will remain indefinable. Therefore, thinking of the factor of safety as a factor of ignorance is probably a better way to consider the nature of the risk level calculations being performed. To that extent, a factor of safety of 1.10 may be adequate for the purpose of maintaining a reasonable balance between public and environmental safety, and the costs of analysis and design and the economies of construction, operation and closure of the containment facility. In general, the less that is known, the less data collected or the less sophisticated the computational analysis the higher the FOS should be set to assure suitable life cycle performance. Therefore, for a FOS 1.10 it is assumed that the engineer has site specific engineering / material test data regarding the nature and strength of materials (cohesion and F angle) for foundational and structural components of the waste facility.

The Department requires that a seismic deformation analysis be performed to assure that all landfill related containment structures are able to resist a permanent displacement, when the pseudo-static analysis results in a FOS less than 1.10. A permanent deformation analysis is to be preformed because it provides a direct estimate of the performance of the landfill structures while the pseudo-static is an indirect estimate. The Department has chosen to use the method suggested by Kvazanijan (1998)[13]. These Maximum Allowable Permanent Cumulative Earthquake Displacement Limits MAPDLs are shown in table 2..

Table 2.

Maximum Allowable Permanent Cumulative Earthquake Displacement Limits

Containment Component	MAPDL	
	Metric	English
Base (Primary) Liner and Foundation	150 mm	5.9 in
Cover Liner	300 mm	11.8 in
Solid Waste Mass	1 m	39.3 in
GCL	100 mm	3.9 in

Simplified Seismic Deformation Analysis – Bray[14] (1998) **(SSDA) -** describes a simplified seismic design procedure for the primary cover liner components of solid waste landfills. The reference steps the designer through

[13] Kavazanjian, E. Jr., 1998 "Current Issues in Seismic Design of Geosynthetic Cover Systems," *Proc 6th International Conference on Geosynthetics*, Atlanta, GA, Vol. 1, pg.219-226.

[14] Bray, J.D. Rathje, E.M., Augello, A.J. and Merry, S.M. 1998, "Simplified Seismic Design Procedure for Geosynthetic-Lined, Solid-Waste Landfills", *Geosynthetics International*, Vol. 5, Nos. 1-2, pp.203-235.

a Screening Analysis[15] to quickly evaluate the potential for seismically induced permanent displacements that are less than or equal to 5.9 inches and 11.8 inches for the base and cover liners, respectively. Then a simplified procedure is provided to step the designer through the characterization of the design earthquake ground motion[16], by estimating (MHA, T_m and $D_{5\text{-}95}$); and develops seismic loading criteria[17] ($MHEA_{base}$, MHA_{top}, V_s, and T_s and the T_m/T_s ratio). Thereafter, the method shows how to calculate the seismic stability in two ways[18];

- Method A is a seismically induced permanent deformation analysis that determines (K_y, U_{base} and U_{cover} (where U is the permanent displacement of the base liner and the cover sequence, respectively.);
- use Method B which calculates a FOS of 1.25 when computing R_C (Cover) and R_B (Base) in a pseudo-static analysis).

Finally the method evaluates the seismic stability of sensitive containment components for the estimated displacements[19].

2-D Seismic Deformation Analysis – Numeric Modeling Methodology (2DSDA) - Although there are an increasing number of numeric deformation models available, they derive from Newmark[20] (1965). Newmark Sliding Block Analysis – A procedure developed by Newmark is typically used to estimate permanent seismic deformation in earthen embankments. This procedure appears to be appropriate for landfills where there is little or no degradation in the sliding mass, shear strength during the dynamic shaking.[21] The following assumptions are made in the Newmark Sliding Block analysis.

- Slide mass is a rigid plastic body
- Displacement does not occur at accelerations below the yield acceleration ($A_Y = K_Y \times g$)

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[15] Ibid, page 223-4, Section 3.2
[16] Ibid, page 224,, section 3.3.1
[17] Ibid, page 224, Section 3.3.2
[18] Ibid, page 225, Section 3.3.3
[19] Ibid, Page 225, Section 3.3.4
[20] Newmark, N., 1965. "Effects of Earthquakes on Dams and Embankments," Geotechnique, Vol. 15, No. 2, pp. 139-160.
[21] Krahn, J. 2004. "*Dynamic Modeling with Quake / W – An Engineering Methodology*," Geo-Slope International, Ltd. 2004, pp. 159.

- The sliding mass deforms plastically along a discrete basal shear surface when A_Y is exceeded
- Static and dynamic shearing resistances are identical
- The effects of dynamic pore-pressures are ignored. The materials do not lose strength during shaking.
- A_Y is not strain dependent and remains constant throughout the event analysis

This procedure is not applicable in situations where there is a significant potential for large (>15%) loss in shear strength due to either the generation of excess pore-pressure or the collapse of the soil grain structure as may be the case for loose silty and sandy soils.[22]

The choice of a 2-D Seismic Deformation Modeling program involves combining the specific site characteristics and the models ability to consider these conditions, such as the immediate impact during shaking of:

- Inertial forces;
- Potential for excess pore-water pressure generation;
- Soil / waste shear strength reduction potential; and
- The delayed impact of these forces on the foundation and waste masses following the event.

The Department must receive adequate documentation and model verification from the designer to assure that the modeling results are appropriate for the physical setting, and provide reliable predictive capabilities given sufficient data, and reliability.

3.6. Special Failure considerations

Liquefaction - The engineer must evaluate the liquefaction potential of the facility and determine the potential for liquefaction. The Department

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[22] Kramer, S. L., 1996. *Geotechnical Earthquake Engineering*, Prentice Hall, pg. 437.

recommends the use of USEPA 600R-95/051 (1995)[23]. This iterative procedure is followed by an increasingly more rigorous procedure for developing an opinion on the probability that the site is susceptible to liquefaction:

- Initial Screening
    - Geologic Age and Origin
    - Fines Content and Plasticity Index
    - Saturation
    - Depth below Groundwater
    - Soil Penetration Resistance
    - If 3 or more of these 5 criteria indicate liquefaction is not likely the potential is considered small, and no further consideration is made, otherwise perform a;

- Liquefaction Potential Assessment (Simplified Procedure) from USEPA[24] (1995) involves intrusive in-situ sampling and analysis to compute a Factor of Safety for Liquefaction (FS_L), if the FS_L is considered unsatisfactory consider performing a; and
- Liquefaction Impact Assessment which is a more rigorous evaluation outlined in USEPA[25] (1995).

Infinite Slope Failure Analysis (Geosynthetics) - The stability of final covers is normally determined using an infinite slope analysis. An infinite slope analysis applies to the slopes where the thickness of the sliding mass is small compared to the slope length. The analysis assumes that movement of the sliding mass will occur parallel to the slope. The forces causing the movement are due to the weight of the slope materials and seepage forces. The forces resisting movement are usually provided by the geosynthetic and /or soil interfaces. The infinite slope equation ignores toe buttressing forces and the tensile strength of the geosynthetic components.

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[23] U.S. EPA 1995, RCRA Subtitle D (258) "Seismic Design Guidance for Municipal Solid Waste Landfill Facilities," Office of Research and Development, Washington DC, April 1995, Pages 74-80.
[24] Ibid, Page 76 and Appendix A
[25] Ibid, Page 80

- $FOS = [\ (C_i/(\gamma z \cos^2\beta)) + (\tan\phi_i(1-\gamma_w(z-d_w)/(\ \gamma z))](\tan\beta)^{-1}$ where FOS is the Factor of Safety, C_i is the interface cohesion, ϕ_i is the interface friction angle, γ is the unit weight of the cover material, z is the depth to the interface, γ_w is the unit weight of water, d_w is the depth to water, β is the slope angle and u is the pore pressure with is expressed as $(z-d_w)$.

Field observations and computations indicate one of the most important factors in the analysis is the depth of water in the slope. The installation of a drainage layer or drainage composite will help mitigate the destabilizing effect by reducing the occurrence of excessive pore water pressure in the slope.

3.7. Material Property Characterization

The density and strength of materials and interface strength values used in the analysis of slope and seismic stability are perhaps the most critical of all the dependent variables. It is crucial that these values be representative of the materials present beneath and used within the facility. Because of the nature of these variables, obtaining site-specific data will be at least time consuming and expensive and in some cases may be nearly impossible to obtain. What follows is a discussion of the type of data that are critical to the analysis process and a presentation of relatively conservative default values for the engineered soil and geosynthetic materials often found in landfill containment construction.

Material Testing Methodologies that shall be employed to obtain appropriate and representative test data for the component parts of the waste containment facility; including earthen materials, geosynthetics, and the solid waste itself:

3.7.1 **Earthen materials** include cohesive and cohesionless soil materials present in the foundation support and component parts.

3.7.1.1 **Cohesive soils** are normally employed in the construction of the primary liner, and may be used as daily, intermediate and final cover.

Classification - Further, a descriptive classification including USCS classification with Atterberg Limits and grain size distribution with hydrometer analysis are useful in classifying the material to assure proper identification and use during construction. These tests are routinely performed by geotechnical testing laboratories and are relative inexpensive to perform.

Density (γ)- Density is important to understand from the viewpoint of both classification of the materials and their impact as drivers in the slope and dynamic failure mechanisms being evaluated. The various density measurements needed include the following: in-place, re-compacted, dry, at field moisture content and saturated.

Strength testing should be sufficient to develop a representative of the material as it will exist beneath and within the structure.

Undisturbed Soils- In the case of foundational materials where the material will not be disturbed by construction but will nevertheless be impacted by supporting the facility, relatively undisturbed representative samples of the foundational soil must be obtained.

Remolded (or re-compacted) Soils – For soils used to construct the primary containment structures, liners, cover materials and embankments for storm water retention and leachate containment structures, remolded specimens of representative bulk samples of the borrow material must be obtained and tested to determine the appropriate moisture-density relationship (typically Standard Proctor ASTM D-698).

Both undisturbed and remolded soils must be tested for cohesiveness and angle of internal friction (c and ϕ) and a multipoint peak and residual strength Mohr Failure Envelope for the appropriate confining pressures must be derived. Typically this testing would be a consolidated – undrained (CU) with pore pressure measurement using triaxial compression or direct shear methodology. Remolded specimens would be prepared for testing at the moisture content(s) and densities consistent with the range required to meet the required liner permeability requirements.

3.7.1.2 **Cohesionless (granular) soils** are normally used in construction of the drainage or protective layers above the primary liners. They may if abundant be used as cover or as structural subgrade. Important data include:

Classification: Similar to cohesive soils except Atterberg Limits are not performed.

Density (γ)- Similar to cohesive soils except in place density and strength parameters of undisturbed soils are empirically estimated via the Standard Penetration Test (SPT), or other more modern technology like the dynamic cone penetrameter.

Strength Testing (c and ϕ) of cohesionless soils may be determined using direct shear methodology using multipoint Mohr Failure Envelope for the appropriate confining pressures. Typically this testing would be a consolidated – drained (CD).

In every circumstance, the laboratory shear strength testing regimen must fully include the range of pressures anticipated within the containment facility, i.e., the ϕ_{max} (Maximum normal load for the testing setup) must be at least as great as the possible anticipated load (load of waste plus all cover and liner components, etc.).

3.7.2 **Geosynthetics** - Interface Shear Strength should be determined using direct shear methodologies for each of the materials that will be placed in contact with the particular geosynthetic. Manufacturer's test data are typically available for contact with high clay content soils, granular drainage layer soils and other geosynthetic surfaces (textured and smooth). These data, while not site specific, are normally a reasonable indicator of the material performance, and may be used if the infinite slope analysis FOS does not approach the limiting FOS. In this case or in the case of a high risk site, actual site-specific testing may be necessary to provide the necessary degree of comfort to assure a reasonable FOS.

3.7.3 **Solid Waste** – Critical parameters are density (γ), and strength of material values (c and ϕ). These parameters normally tend to increase with time as the primary consolidation of the waste mass takes place. Typically the primary consolidation is nearly complete at closure and secondary consolidation continues thereafter. Development of site-specific data on the waste mass is difficult and very expensive to achieve, and is rarely done unless the risks at the site warrant it.

Waste density can be approximated from literature and regional experience of the nature of the waste material, filling and waste compaction operation. Ultimately, density data can be gleaned from used airspace versus tipping tonnages. Since density is a failure driving factor, it is considered prudent to overestimate the density rather than to underestimate it when evaluating stability of the waste mass.

Representative strength parameters are much more difficult to obtain as they rarely become known unless a slope failure within the waste mass is manifested and strength parameters are back-calculated. Therefore, it is common practice to estimate the strength of the waste using literature references.

The use of default values such as strength and density values as input in lieu of site specific material properties as input into the pseudo-static analysis will result in raising the minimum allowable factor of safety threshold that necessitates a deformational analysis from 1.10 to 1.25.

Table 3.

Worksheet for slope stability and seismic deformation analyses

Material Description	Density γ (pcf)	Cohesion c (psf)	Angle of Internal Friction φ	Reference Source
Vegetative Cover (Cohesive)	102	1200-1600	25-45	[26, 27]
Final Cover Clay Liner	92 - 96	1400 - 2800	14 - 24	[28]
Interim Cover (Cohesive)	90 -102	50 - 2800	14 - 20	[29, 26, 27]
Solid Waste (Municipal)	30 - 65	50 - 2000	1 - 37	[30, 31]
Municipal Baled	32 - 47	50 - 2000	1 - 37	[30, 31, 32]
Geosynthetic Drainage Composite (GDL)		40 - 60	19.6 – 26	[33, 34, 35]
Geomembrane (Textured) (Cover)	75 - 80	40 - 60	15 – 32*	[33, 34, 36]
(Primary Liner)		20 – 520	7 – 35*	[28, 33, 34, 36]
Geomembrane (Smooth) (Cover)		-	6 – 9*	[33, 34, 35]
(Primary Liner)		-	4 – 6*	[33, 34, 35]
Geosynthetic Clay Liner (GCL)				
Granular Layer	110 - 124	0 - 800	26 - 40	[26, 28, 29]
Primary Clay Liner	92 - 96	1400 - 2800	14 - 24	[26, 28]
Foundation Soil Clay	90 - 100	1000 - 3000	14 - 24	[26, 28]
Silt (Soft to Medium)	100	0 - 1500	15 - 20+	[27, 37]
Sands and Gravels (Loose to Dense)	90 - 140	0 - 800	25 - 48	[26, 28, 29, 37]
Rip Rap / Rock Fill	145	-	30 - 50	[29]
Bedrock (Impenetrable)	NA	NA	NA	[38]
* values indicate interface contact with non-woven geotextile				

[26] Spangler, M.G. & Handy, R.L., Soil Engineering, 3rd Edition, Intext International, 1973, pp. 559, Table 22-1

[27] Seelye, E.E., Design-Data Book for Civil Engineers, 3rd Edition, J. Wiley, 1968, pp 9-08, Tables G & H

[28] Abramson, L. W. & Lee, T.S. et al., Slope Stability and Stabilization Methods, 2nd Edition, J Wiley, 2002, pp. 42. Table 1.8

[29] Huang, Y,H., Stability Analysis in Earth Slopes, Van Nostrand Reinhold, 1983, pp. 35-36, Table 3-1 & 3-2

[30] Sharma, H.D. & Lewis, S. P., Waste Containment Systems, Waste Stabilization, and Landfills: Design and Evaluation, 1st Edition, J. Wiley, 1994, pp. 65-66 & Figure 2.15

[31] Abramson, L.W., Slope Stability and Stabilization Methods, 2nd Edition, J. Wiley, 2002, pp 678-679, Table 10.2; Figure 10.9; pp. 687-688, Table 10.9

[32] Montague, D.J. and Baker, J.T., 1998, Baling Out Small Landfills, Waste Age Magazine, April 1, 1998

[33] GSE Technical Note – Direct Shear & Friction Angle Testing for GSE Geomembranes (TN018 R11/04/02, www.gseworld.com)

[34] Sharma, H.D. & Lewis, S. P., Waste Containment Systems, Waste Stabilization, and Landfills: Design and Evaluation, 1st Edition, J. Wiley, 1994, pp. 147 - 149 & Tables 3.13 – 3.15

[35] Fox, P.J. and Stark, T.D., State-of-the-art report: GCL shear strength and its measurement, Geosynthetics International, 2004, Vol. 11, No. 3, pp. 141 - 175

[36] Stark, T.D., et al., M ASCE, HDPE Geomembrane / Geotextile Interface Shear Strength, Journal of Geotechnical Engineering, March 1996, pp 197 - 203

[37] Merritt, F.S. et al., Standard Handbook for Civil Engineers, Fourth Edition, McGraw Hill, 1996, pp 7-27 Table 7.7 and pp 7.81 Table 7.15

[38] Krahn J., Stability Modeling with SlopeW, An Engineering Methodology, First Edition, May 2004, Geo-Slope International, Ltd., Calgary, Alberta, Canada, www.geoslope.com, pp 141

4.0 Technical and Administrative Content of the Seismic Evaluation

4.1. Technical Content

Slope Stability Analysis – is dependent on site conditions, material properties, and construction means and methods, and each slope stability analysis should be approached in a manner that accounts for the site-specific conditions. As such, a rigid approach in the data required and the analysis used is not realistic, but in general terms the following should be considered the minimum scope necessary to suitably evaluate the global slope stability of a landfill and its containment components. The design engineer should provide the Department with the rationale, cross-sections and plan views for the critical slopes that may occur during the following three landfill phases:

- Excavation and construction of the primary containment systems
- Operation and filling of waste
- Closure and post closure activities.

The rationale for the selection of soil materials, and geosynthetics must include:

- Detailed information from site-specific subsurface exploration which depicts not only the configuration of the foundational geology, along with complete information on the strength of materials and location and presence of groundwater.
- Relatively undisturbed (peak and residual) shear strength of material strength data (c and ϕ) for foundational soil materials that will remain in place during and after construction of the containment facility should be provided.
- Re-compacted (peak and residual) shear strength of material data (c and ϕ) for soil materials to be employed in the construction of the landfill system components, such as primary clay liners, drainage or protective layers, daily, interim and final covers
- Interface strength characteristics (c and ϕ) of geosynthetic materials and the corresponding soil materials they will contact

Rationale for the use of density (γ), and strength of material values (c and ϕ) for the solid waste fill materials anticipated at the landfill complex, e.g., municipal, baled, C & D, or industrial mono-fill or special waste.

The rationale for the choice in the computational methodologies (Spencer, Morgenstern-Price or Sharma are typically considered most appropriate) used to determine the slope stability FOS.

Summary of input parameters shall include:

- x and y coordinate sets for all material region model boundaries;
- Material properties (γ, c and ϕ) and any legend keys needed to understand the graphic output (similar to the table of default values provided above);
- phreatic water surface coordinates or pore water pressure inputs if applicable and
- a presentation of any physical calculations and / or computer output for critical slip surface(s) conditions for each of the three landfill phases discussed above. This includes determination of critical slip surfaces for both static and dynamic (pseudo-seismic) cases for PSSSA or for determination of A_y when performing the SSDA or the 2-DSDA noted above.

Pseudo-Seismic Slope Stability Analysis – is dependent upon the slope stability analysis previously described except for the addition of the appropriate seismic coefficient ($k_y = k_{vertical} = k_v$ and $k_x = k_{horizontal} = k_h$). These values are assigned using the most current National Seismic Hazard Mapping Project map depicting the Peak Acceleration (%g) with 10% Probability of Exceedance in 250 years (or equivalent i.e., 2% in 50 years. Typically, the analysis is performed by assigning the K_x value as the %g shown for the site location. A more conservative approach is to also assign both the K_x and the K_y this value to determine which scenario produces the lowest FOS. It is relatively simple to determine the appropriate Peak Ground Acceleration (PGA) given the input of the site latitude and longitude by using the USGS web site which interpolates the previously referenced map information[39].

Seismic Deformation Analysis may be either the Simplified – Contained in Bray[40] (1998) or the 2-Dimensional – Site Specific which typically includes all the special geometries and strength of material information used in the SLESSA and PSSSA, plus the acceleration vs. time record of the design earthquake or a acceleration vs. time record of the representative ground motion for the region.

4.2. Administrative Content

The engineer must clearly identify and present each critical section analysis for each containment system phase, i.e., Construction Phase, Filling Operations Phase and Closure/Post Closure Phase.

- include in tabular form the complete x and y coordinate sets for all boundary input points for each material type considered in the stability analysis,
- include in tabular form in the text of the report, the material region number, material name, dry and wet density (unit weight) in pcf, and (c and ϕ) strength parameters (peak or residual as appropriate), as well as the justifying reference (literature or test data).
- include for each analysis graphic report critical section, a graphic output showing all material regions with the corresponding color and material data table including material region number, material name, strength parameters, density, and parameter failure soil model (i.e., Mohr-Coulomb).
    - In addition each graphic should clearly identify the project name, critical section name and number if applicable, date of run, Analysis Method, and Seismic Coefficient Values (K_v and K_h).

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[39] http://eqint.cr.usgs.gov/eq-men/html/lookup-2002-interp.html

[40] Bray, J.D. Rathje, E.M., Augello, A.J. and Merry, S.M. 1998, "Simplified Seismic Design Procedure for Geosynthetic-Lined, Solid-Waste Landfills", *Geosynthetics International*, Vol. 5, Nos. 1-2, pp.203-235.

- If a Grid and Radius method of analysis is performed, include the grid points as well as the FOS contour map over the grid system with contour labeling that clearly indicates that the minimum FOS is well within the range of the grid boundary.
- The critical slip surface must be depicted without color (clear mode) showing the location of the vertical slices analyzed superimposed over the material regions.
- Phreatic surfaces must be depicted if employed in the modeling run.
- The engineer must provide a SLOPE/W (and QUAKE/W, SIGMA/W or SEEP/W) XXX.grz files for each critical case to allow WDEQ the opportunity to run the analysis on the free viewer licensed software.
- The engineer should provide output showing the FOS mapping (for the SLESSA and PSSA output) of the critical section as an additional report element to assist the Department in understanding the nature of the failure result across the entire critical section.

Technical Report – A technical report that develops the rational and clearly presents the computational logic that follows from data input through modeling output must accompany the technical modeling output mentioned previously. The format may vary, but the following format is recommended:

- Introduction
- Project Information
  - Critical Sections
  - Spatial configuration
  - Material Properties of Soil, Waste and Foundation
- Slope Stability Modeling Results
- Seismic Analysis
  - Ground Motion Parameters
  - Seismic Properties of Containment System and Foundation

- FOS computation / or Deformation Analysis
- Methodology / Theory
- Sensitivity analyses on the most critical variables
- Modeling Verification
- Supporting Appendices
- References
- Model Theory and Verification presentation – The engineer should include a discussion of the computational methodologies employed in sufficient detail for the Department to understand the technical basis. The methodology may be summarized in the text with details provided in the appendices. Similarly, verification of the model is the process by which the model has been shown to produce consistent accurate results when compared to know performance benchmark cases. This needs to be discussed and sufficient detail must be provided to the Department to provide assurance of the efficacy of the model. In the case of the methodologies and verification, the level of detail may vary depending on the Model and methods used. The engineer should consult with the Department on the level of detail needed in advance.

5.0 Glossary of Terms

The purpose of this section is to define the terminology used throughout this guidance document and to provide consistency with the terminology used in the SWRR.

Lithified Earth Material - all rock, including all naturally occurring and naturally formed aggregates or masses of minerals or small particles or older rock that formed by crystallization of magma or by induration of loose sediments. This term does not include man-made materials, such as fill, concrete and asphalt or unconsolidated earth materials, soil or regolith lying at or near the earth surface.

Seismic Impact Zone – area with a ten percent or greater probability that the maximum horizontal acceleration in lithified earth material, expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10g in 250 years.

Maximum horizontal acceleration in lithified earth material - means the maximum expected acceleration depicted on a seismic hazard map, with a 90 percent or greater probability that the acceleration will not be exceeded in 250 years, or the maximum expected horizontal acceleration based on a site –specific seismic risk assessment.

Static Slope Stability Analysis – An engineering computation concerned with identifying critical geological, material, environmental and economic parameters that will affect the constructed project, as well as understanding the nature, magnitude and frequency of potential slope problems.[41] The aims of slope stability analysis in this guide are:

- To assess the stability of slopes under short-term (often during construction) and long –term conditions.
- To evaluate the effect of seismic loadings on slopes and embankments.

Pseudo-Seismic Slope Stability Analysis – The pseudo-seismic slope stability analysis is the simplest approach to evaluating the stability of a slope in a seismic impact zone. The pseudo-static seismic analysis employs the limit equilibrium method that is modified to include horizontal and vertical static seismic forces that are used to simulate the potential inertial forces due to ground accelerations in and earthquake.[42] These seismic forces are assumed to be proportional to the weight of the potential sliding mass times the horizontal and vertical coefficients, K_h and K_v, as expressed in terms of the acceleration of the underlying earth (in units of g).

Peak Horizontal Ground Acceleration – The peak horizontal ground acceleration is the maximum acceleration along the horizontal axis that would probabilistically be experienced at the base of the considered facility throughout the entire duration of the ground motion event.

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[41] Abramson, Lee W. et al, *"Slope Stability and Stabilization Methods"*, 2nd Ed., John Wiley & Sons, Inc., 2002, pp. 2
[42] Ibid, Page 394

Seismic Deformation Analysis – an engineering evaluation by which a probabilistic estimate of the magnitude of seismically induced permanent deformation of the containment structures and corresponding waste mass is made for comparison against the mandated maximum deformation thresholds.

Liquefaction – The complete loss of strength typically of deltaic or unconsolidated granular (non-cohesive) soils that occurs as a very rapid normal consolidation due to the loss of grain-to grain contact in the presence of excess pore water pressure as triggered by vibration (typically from machinery and earthquakes).[43]

Geosynthetics – is a term used to describe a range of almost exclusively man-made, polymer products used to solve geotechnical problems. The term is generally regarded to encompass four main products: geotextiles, geonets / geogrids, geomembranes and geocomposites. In this guide geosynthetics are primarily employed in waste containment, fluid (infiltrated water or leachate) drainage conveyance, and slope stabilization of the waste mass or waste containment components.

Solid Waste – includes municipal, industrial solid waste and construction and demolition debris as defined by statute and regulation of the Wyoming Department of Environmental Quality.

6.0 Approval

I have reviewed and approved the recommendations described in this guidance document.

______________________________ Date: 7 Dec 07

LeRoy C. Feusner, P.E., BCEE.
Administrator
Solid & Hazardous Waste Division

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[43] Spangler, Merlin G. and Handy, Richard L., "*Soil Engineering*", 3rd Ed., Intext Educational Publishers, 1973, pp. 397